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Coastal Engineering Technical Note

EFFECTS OF REFLECTIVE WALLS IN HARBORS - A CASE STUDY

PURPOSE: To provide an example of operational problems at a loading facility exacerbated by the placement of reflective retaining walls.

INTRODUCTION: The reflection of surface waves from vertical walls and the resultant standing wave pattern are well-known phenomena in coastal engineering. The use of vertical sheet pile retaining walls, as opposed to rubble revetments, in the construction of harbor facilities is commonplace because it allows maximum draft adjacent to the shore facilities at comparitively low cost. The underlying assumption in designing a vertical wall is that the incident wave energy is sufficiently low during acceptable operational windows that the vessels and loading facilities can tolerate the incident plus reflected wave energy.

Data were collected on wave conditions at Burns Harbor, Indiana, at the South end of Lake Michigan (Figure 1) under the Monitoring of Completed Coastal Projects (MCCP) Program. The harbor, constructed in 1968, consists of an attached, L-shaped rubble-mound breakwater protecting an outer harbor and two arms formed by reclaimed land behind rip-rap revetments and steel bulkheads. All of the vertical bulkheads are aligned North-South with the exception of a rectangular grain dock, built at a later date, projecting front and center from the North Wharf. Wave gages were placed outside of and behind the breakwater between December 1985 and June 1988, and in front of the grain dock from January to June, 1987.

Background: The breakwater was constructed using cut limestone blocks and a relatively low core. This produced a relatively permeable structure, with a wave transmission coefficient, Kt, on the order of 25 percent. A number of incidents involving damage to ships, including two sinkings, have prompted complaints from the harbor customers, particularly from the grain dock operators. The cause most often cited in the complaints has been "excessive transmission" of waves through the breakwater.

Two physical model studies were conducted in the design phase of the harbor. A three-dimensional (3D) physical model was tested to optimize the geometry of the entrance and the harbor layout (UF, 1964). This model used an impermeable breakwater, and thus only simulated wave energy coming through the entrance. In any event, the grain dock was not included in this model; the North Wharf had a revetted slope.

The maximum design wave used in the 1964 3D model was a 10-ft, 9-second wave from the North. The model predicted wave heights in front of the North Wharf would be approximately 10 percent of the incident height.

A larger scale, two-dimensional (2D) model was used to evaluate breakwater stability and wave transmission (Jackson, 1967). This study was conducted in a flume, and measured waves in front of and behind the structure. It predicted a wave transmission coefficient of 24 percent for a somewhat higher design condition - 13 ft at 11 seconds - and 32 percent for the maximum incident wave tested, 18 ft.

Selection of the design wave is always a difficult task, and was accomplished using engineering judgement and the data available at the time. Estimates of the height varied from 10 to 16 ft, but a final value of 13 ft at 11 seconds was agreed upon. Although the designers recognized that the breakwater would allow waves into the harbor under the design conditions, there remained some uncertainty about the extent to which transmitted waves would interfere with harbor operations.

<u>Observations:</u> In 1988, winter storm waves approached to within 90 percent of the original design wave of 13 ft, and in 1987, exceeded it by almost 25 percent. Table 1 lists wave height for several return intervals based on a wave hindcast performed in 1990 by CERC.

TABLE 1

Return Interval - Years	Wave Height -ft
5	14.1
10	15.5
20	16.7
50	18.2

Figure 2 is a plot of wave heights in front of the breakwater vs wave heights behind the breakwater for the 1967 model data and for prototype data measured in 1987. The 1967 2D physical model results underpredict transmission at waves less than 11 ft and slightly overpredict for larger waves. Figure 3 is a similar plot, but using the prototype waves measured in front of the grain dock during the same period. Maximum measured significant wave height is 6.7 ft, which is over 40 percent of the incident wave height.

The wave height measured by a gage in a standing wave pattern depends upon its position. In a true clapotis, a node occurs at a distance of L/4, or about 90 ft for the typical 11-second storm waves observed in the harbor. The gage was placed about half of this distance in front of the grain dock bulkhead to represent the approximate center of a moored barge, resulting in a wave height of about 70 percent of the antinode. This approach leads to an estimated wave height at the wall of about 9.5 ft, which agrees well with an assumption of 100 percent reflection of the "incident" wave (i.e., the 4.7 ft wave behind the breakwater propagating toward the grain dock). The slope of the water surface between the wall and the node, which occurs near the outboard side of moored vessels, will vary from +3 to -3 degrees each wave period. This is sufficient to cause large amplitude motions and place excessive loads on mooring hardware and the vessel alike.

Conclusions: The presence of reflected waves in front of the grain dock is easily predicted. Less obvious is the frequency of occurrence of waves high enough to cause problems. Comparison with modern hindcast techniques indicates that the design values chosen in the 1960's have significantly shorter return intervals than originally believed.

The 3D model did not simulate the actual breakwater permeability or the final harbor geometry, and thus could not adequately predict wave characteristics in the harbor. Even if the grain dock had been included, resulting in complete reflection, the model still would have underpredicted the measured wave heights by a factor of about 2.

The 2D model, though run with monochromatic waves, provided accurate predictions of actual random wave transmission values using significant wave heights. Without field measurements, the impression that the breakwater was more permeable than predicted would have remained. Apparently the significance of the predicted transmission coefficient of about 25 percent was not appreciated by the port's users, nor fully considered by the designers of the grain dock.

A site-specific problem was created by the design, placement and orientation of a loading facility at the grain dock. Considerable attention has been given to developing modifications to the breakwater to reduce wave transmission. To a first approximation, reduction of the waves at this facility to less than 1 ft height for the ten-year event would require a second breakwater of similar design behind the first. Remedial planning studies should also consider the option of modifying or relocating the grain dock.

ADDITIONAL INFORMATION: Contact Mr. David D. McGehee, P.E., Research Hydraulic Engineer, CERC, at (601) 634-4270.

References:

JACKSON, R.A., 1967, "Stability of Proposed Breakwater Burns Waterway Harbor, Indiana," U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS

UNIVERSITY OF FLORIDA, 1964, "Hydraulic Model Study of Burns Waterway Harbor,"

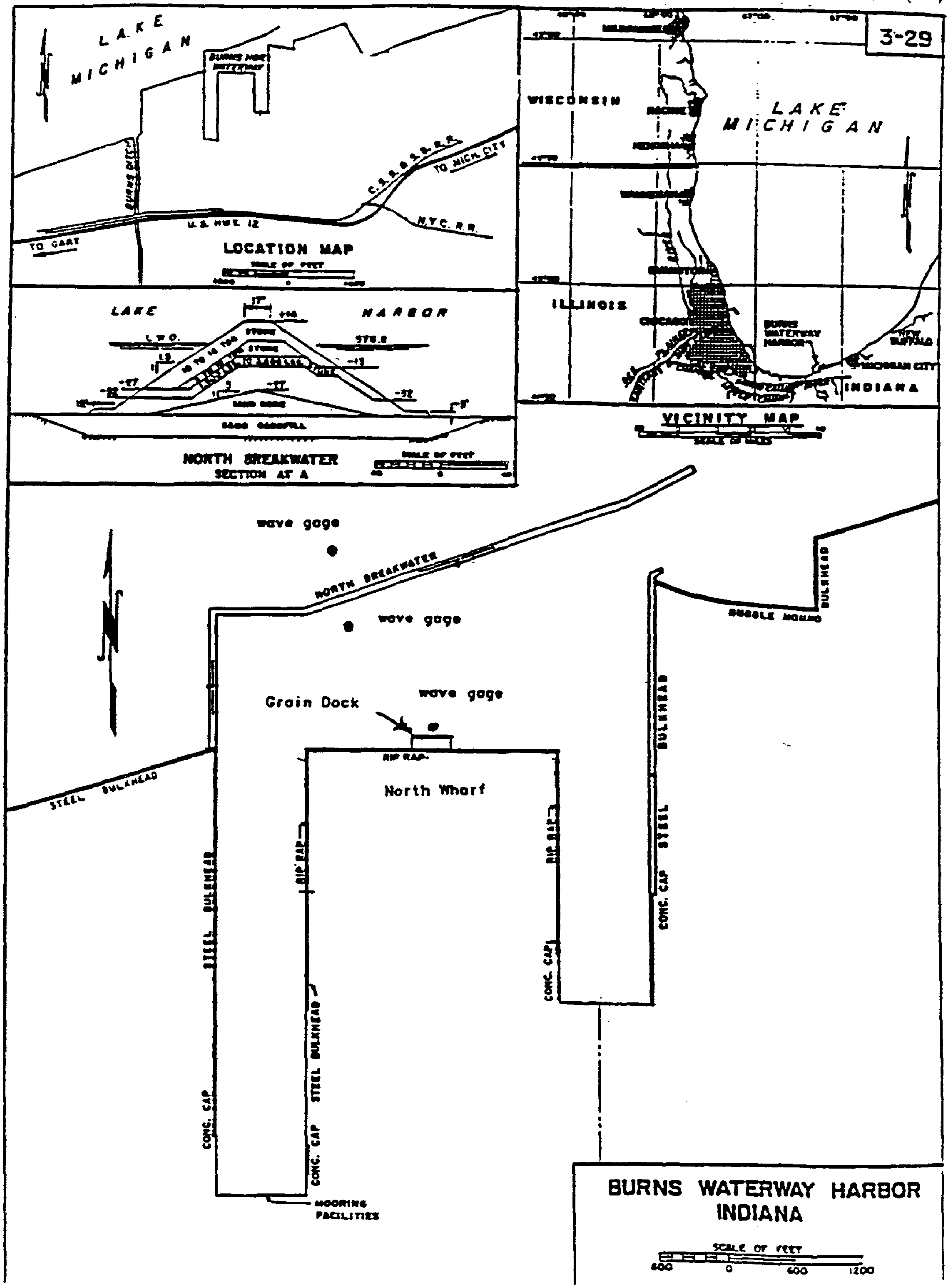

Figure 1 - Site map of Burns Harbor, Indiana

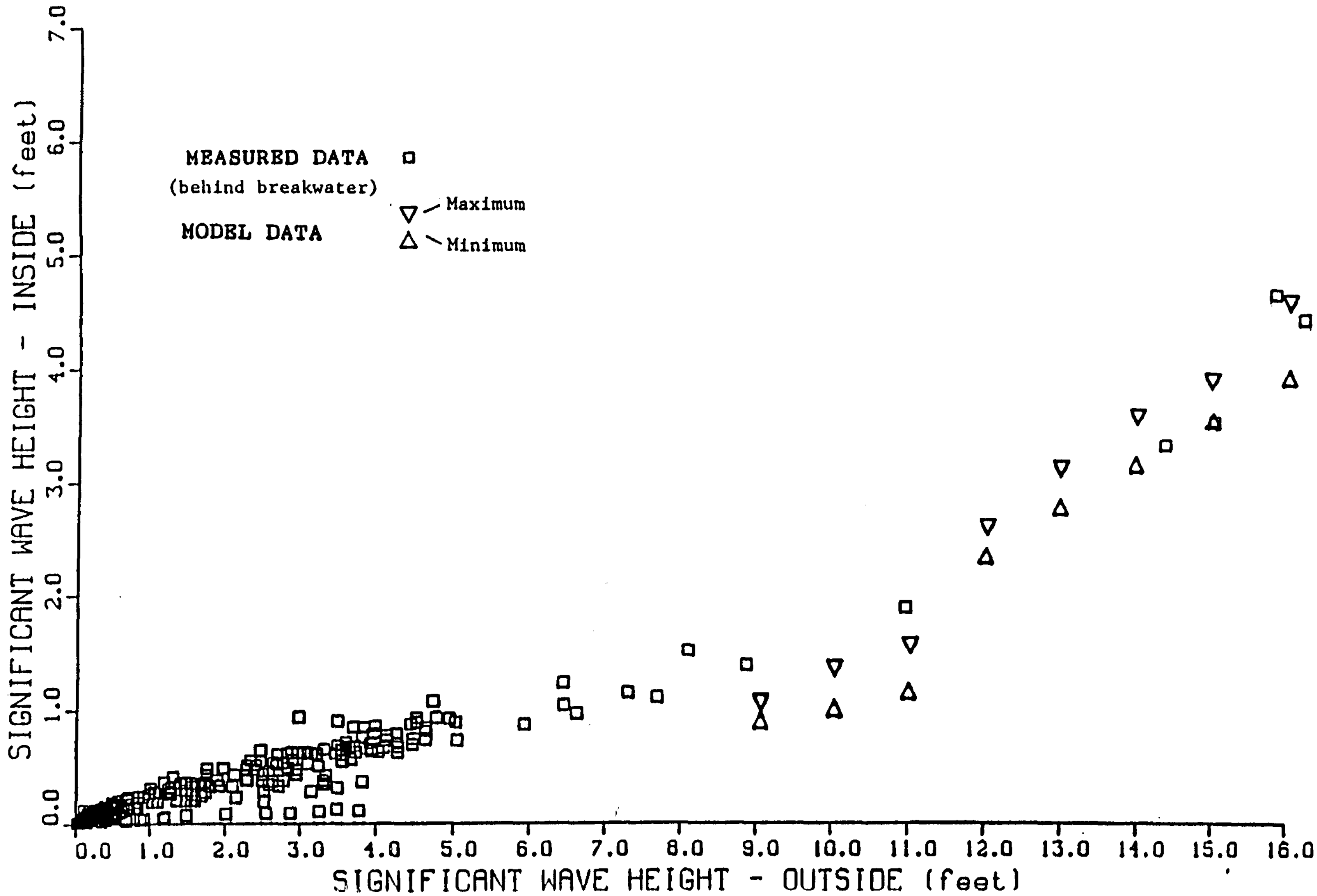

Figure 2 - Transmitted Wave Heights Behind Breakwater

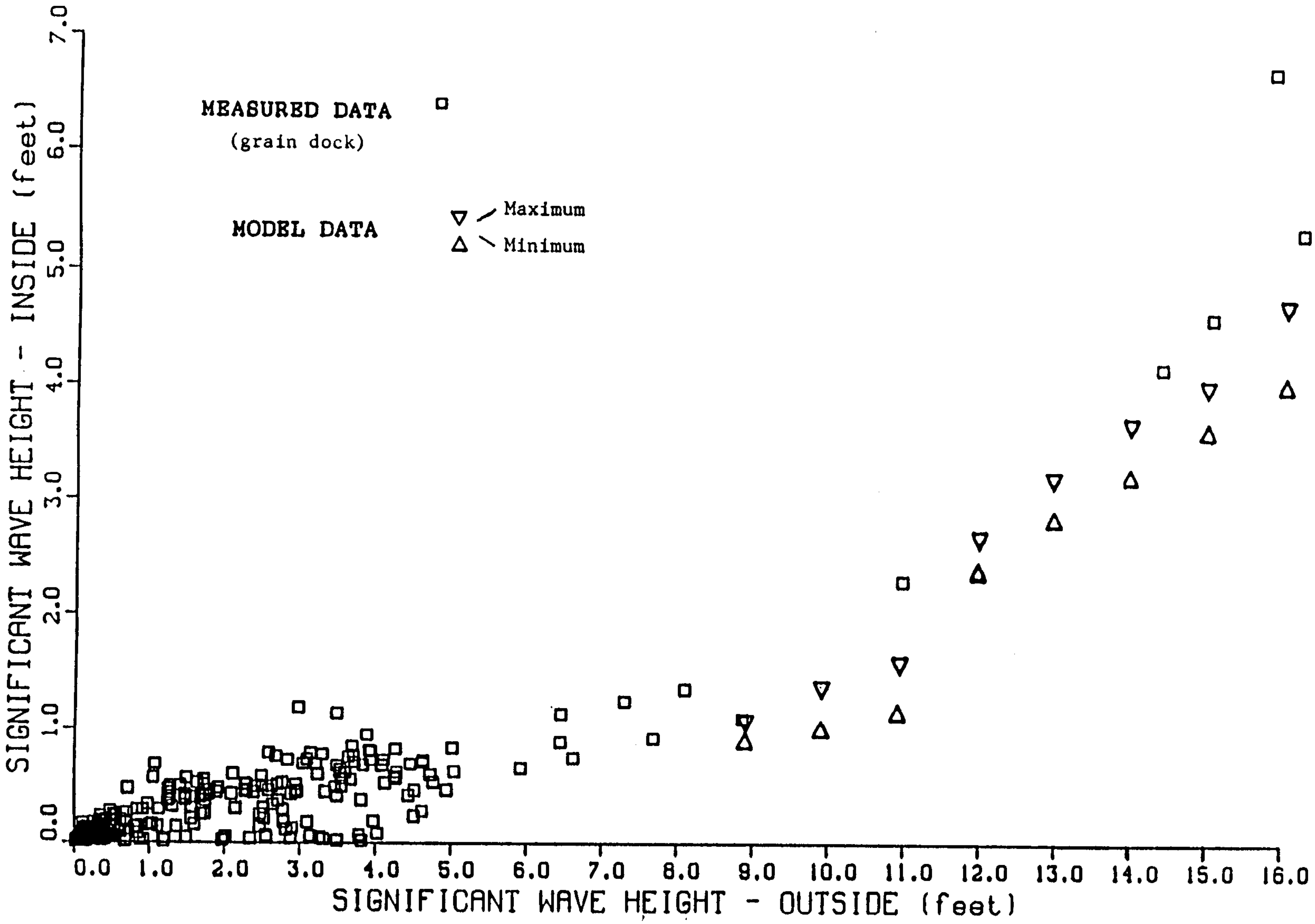

Figure 3 - Transmitted Wave Heights at Grain Dock